

Testing and analysis of masonry arches subjected to impact loads

Paulo B. Lourenço, Tibebu Hunegn and Pedro Medeiros

Department of Civil Engineering, ISISE, University of Minho, Guimarães, Portugal

Nuno Peixinho

Department of Mechanical Engineering, University of Minho, Guimarães, Portugal

ABSTRACT: A prototype single span masonry arch was designed and its capacity was assessed both experimentally and numerically. The geometric design process was based on the dimensional-constraints of the laboratory. A quasi-static experimental test was conducted on the arch and the force deformation behaviour of the arch was obtained. Joint interface properties might play an important role in numerical assessment of masonry stone bridges; therefore the influence of these parameters is addressed. A sensitivity numerical analysis was carried out to figure out the range of values that gives similar global behaviour of the arch from both numerical and experimental simulations. Subsequently, the results of the impact test are given and a numerical simulation able to capture the main characteristics of the response is presented.

1 INTRODUCTION

An explosion on or nearby a bridge can cause catastrophic damage locally or globally leading to the total collapse. Due to the threat from such extreme loading conditions, efforts have been made during the past decades to develop methods of structural analysis and design to resist blast loads, particularly from military defence engineering. However the issue has become serious for civil infrastructures too after the campaigns of terrorist attacks.

Blast analysis of railway masonry bridges involves the evaluation of the structural performance of stone masonry arches, which are the basic structural components of such bridges. Hence the issue involves three major topics:

- (1) Accurate specification of actions on stone masonry bridges arising from explosions. When an explosion occurs, the bridge can be subjected to blast pressure, impacts from fragments and base excitation from ground shock. Hence for accurate specification of such actions it is important to understand the interactions that occur between the structure and its surrounding during explosions;
- (2) Accurate estimation of the resistance curve of the structural system, namely the arches, using either a simplified guidelines or using robust numerical simulations;
- (3) Accurate performance assessment of the bridge against such actions using either simplified procedures or refined numerical simulations.

Here, a prototype single span stone masonry arch was designed. Both dry joint and mortar joint arches were simulated experimentally and numerically to investigate the resistance of the arch. A simplified limit analysis using RING software and an advanced non-linear FEA using DIANA software were carried out to replicate the failure modes and capacity curves of the arch. A displacement controlled quasi-static experimental test was conducted on the arch specimens to calibrate numerical models and to identify and define the bounds of significant parameters that affect the resistance curve.

To characterize the performance of stone masonry arches at high rate loading, a single span stone masonry arch was assessed experimentally and numerically for impact loading. The prototype arches were tested experimentally using a drop weight apparatus. The numerical

simulation was carried out using an explicit dynamic analysis software LS-DYNA. Further details of the tests and analysis are given in Hunegn.

2 STATIC PERFORMANCE OF A MASONRY ARCH

The main purpose of this Section is to assess the quasi-static capacity of a single span stone masonry arch. In order to quantify the dynamic enhancement at structural level, it is necessary to predict the quasi-static force deformation capacity. This gives also a bound to the energy absorbing capacity of the arch, the peak static strength, and the pre-peak and post-peak effective stiffness of the structure. With this aim, a prototype single span masonry arch was designed and its capacity was assessed both experimentally and numerically. The geometric design process was based on the dimensional-constraints of the laboratory. The analysis of the arch was carried out to predict the failure mode in advance of the experimental work. A quasi-static experimental test was conducted on the arch and the force deformation behaviour of the arch was obtained. Joint interface properties might play an important role in numerical assessment of masonry stone bridges and the influence of these parameters is addressed here.

2.1 Specimen and experimental setup

The arch was designed primarily for impact response simulation and the global dimensions of the arch were fixed based on the geometric constraints of the drop-weight apparatus. A dry circular arch in stone with an out-of-plane thickness of 200mm was designed. The specification of the clear span (1200mm), rise (400mm) and thickness (160mm) are primarily based on the laboratory geometric constraints and the ranges set from past experience.

The stone blocks were fabricated from granite rocks of Guimarães, a city in the North of Portugal. Granitic stone is the most widely used construction masonry material of ancient structures here. This is a fine to medium-grained biotitic granite characterized in Vasconcelos et al., with a compressive strength about 100N/mm² and a tensile strength about 5 N/mm², which is influenced by weathering.

The arch was assembled on a reaction steel frame made of the I-section steel girders. The abutments are well restrained laterally against the reaction frame. The girders are 450mm deep and well braced, providing sufficient lateral and vertical constraint, see Fig.1. The construction was carried out using a formwork that fits the intrados of the arch. The assembling of the units starts from left and right abutments, and then stacking each unit one by one up to the key stone. Finally the key stone was driven in between the right and the left rings of the arch.

Once the arch was constructed, a wooden bearing, with horizontal width of 110mm, out of plane thickness of 200mm, and a height (on the left) of 50mm, was glued to the fifth stone block using rapid hardening epoxy resin adhesive. The wooden wedge was placed in such way that it coincides with the location of the quarter span (300mm from the centre line of the arch). More over a steel bearing, with a roller at the top, was placed over the wooden wedge. This enables to make sure that no rotational restraint exists between the arch and the loading cell.

Finally the load cell and the actuator were positioned at the quarter span of the arch; and a total of five external displacement transducers, LVDTs, were set on the arch to measure the deformations. The first and the fifth LVDTs were planned to measure the sliding at the abutment arch interfaces. LVDT2 was used to measure deformation at the loading point. The third and fourth LVDTs were positioned to measure the opening at the rotational hinges on the left and right quarter span. The actuator and the LVDTs were connected to a data acquisition system and calibrated. A single test was carried out with a displacement control rate of 0.005mm/s.

2.2 Results and discussion

The arch was found to carry a peak load of 2.71kN and was able to deform quasi statically up to a vertical displacement of 12 mm, with a reduction in strength of 35%. Fig.2 and 3 illustrate the obtained failure mode, including two rotational hinges at the extrados and intrados of the left and right quarter span, one rotational hinge at the right arch-abutment interface, and one sliding

release at the left quarter span below the loading point. At the abutments, there was also some sliding. The opening of the left quarter rotational hinge stops at around 3.5mm, which is due to the threshold limits of the transducer. A maximum of 10mm opening was measured at the right quarter span hinge. It should be noted that the opening measurements are merely indicative, as the transducers were not accurately fixed at the opening edges parallel to the direction of flow.

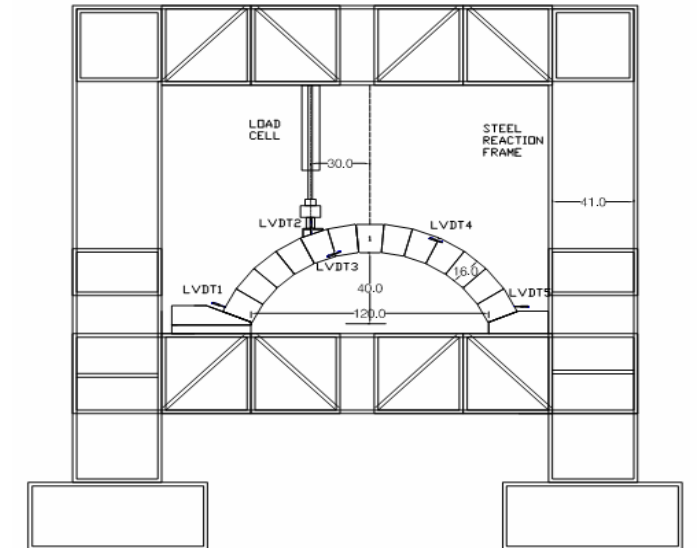


Figure 1 : Test set-up (dimensions in cm).

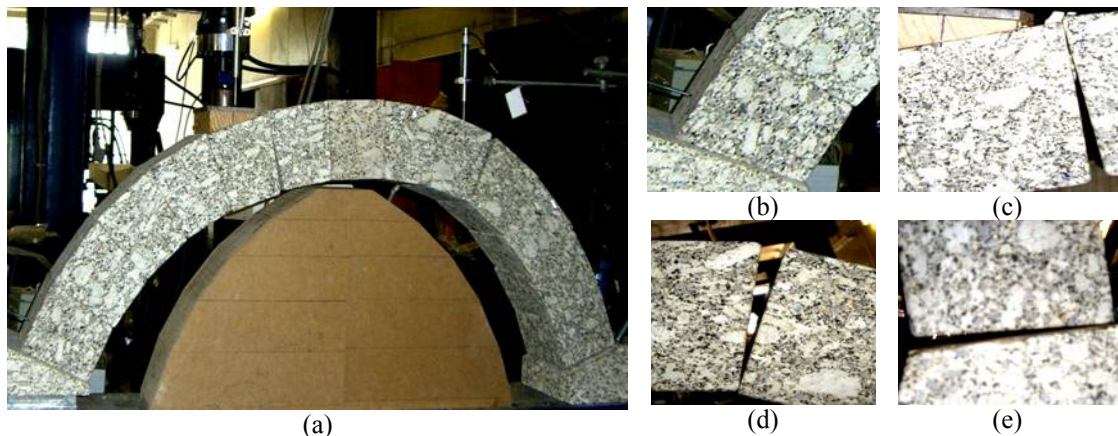


Figure 2 : Deformation at the end of the test: (a) global view; (b) left abutment; (c) left quarter span; (d) right quarter span; (e) right abutment.

Fig.4a shows numerical results using the limit analysis software RING, where it is shown that a similar failure mechanism to the observed one can be obtained, using a coefficient of friction of 0.65 (Vasconcelos, 2005). It was also found that the collapse load was about 50% higher than the experimental value. For this reason, a non-linear finite element analysis was carried out with the mesh provided in Fig.4b. It was found that the pre-peak stiffness and the peak strength of an arch depend on the stiffness of the joints and a value of 5 N/mm³ provided a very good fit with the experimental results, see Fig. 4c. Finally, Fig.4d presents the deformed mesh at collapse, together with the contour of the minimum (compressive) principal stresses. The values obtained for the compressive stresses are rather low in comparison to the strength of the stone, meaning that this parameter has no influence in the results.

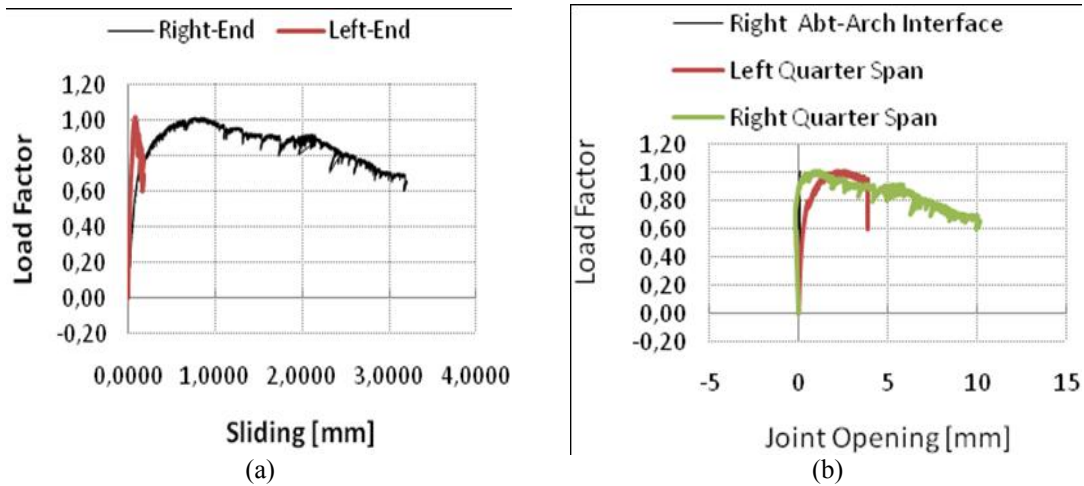


Figure 3 : Measured movements at joints: (a) sliding at arch abutments; (b) gaping at rotational hinges.

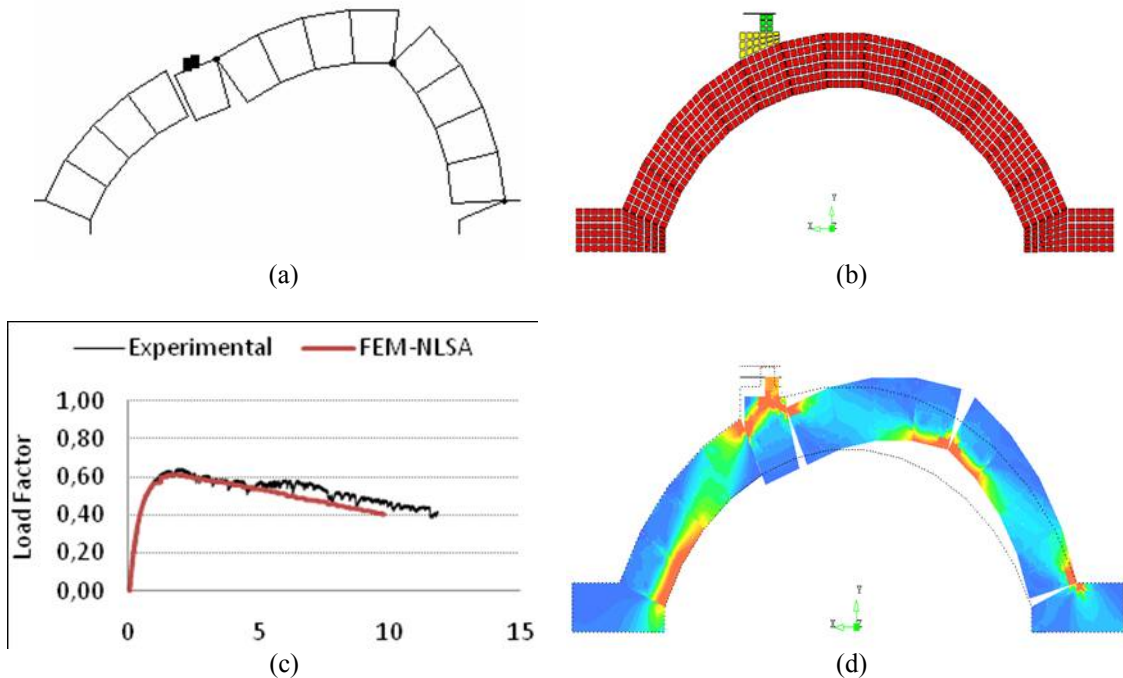


Figure 4 : Numerical results: (a) failure mechanism using limit analysis (RING); (b) finite element mesh (DIANA); (c) force-displacement diagram, for quarter span vertical displacement (in mm), using DIANA (d) minimum principal stresses and failure mechanism using DIANA non-linear finite elements (red indicates maximum compression).

A sensitivity study is shown in Fig.5, in order to discuss the influence of the joint interface stiffness and of the friction coefficient. The enormous influence of the joint interface in the response is shown, both in terms of pre-peak stiffness and failure load. It is also shown that a change in the friction coefficient within reasonable changes has marginal effect in the response.

3 DYNAMIC PERFORMANCE OF THE MASONRY ARCH

To characterize the performance of stone masonry arches at high rate loading, the same stone masonry arch was assessed both experimentally and numerically for impact loading. The prototype arches were tested experimentally using a drop weight apparatus. The numerical

simulation was carried out using an explicit dynamic analysis software LS-DYNA. Finally a calibration of the numerical model was made using the experimental results.

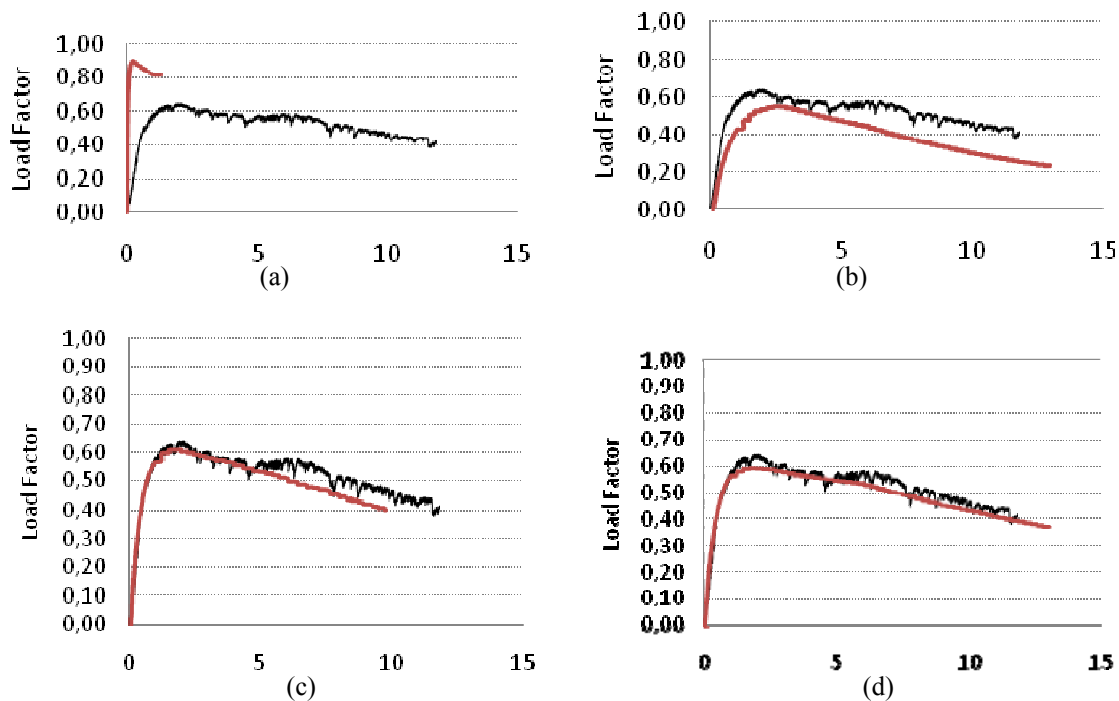


Figure 5 : Numerical sensitivity results with DIANA: (a,b) stiffness change of the interface from infinity to 2.5 N/mm²; (c,d) friction coefficient change of the interface from 0.7 to 0.6.

3.1 Experimental setup and results

A series of eight prototype arches were assembled within the drop weight apparatus for impact test, with the results shown in Table 1. The first six arches were dry jointed and the last two were mortared jointed. The arches were constructed using always the same stone units, which were the same used for the quasi-static test. The setups were labelled to refer the amount of drop weight and drop height. For example M60H300 refers a test specimen subjected to an impact from mass of 60.93kg dropping from a height of 305mm.

Table 1 : Impact test setups and observed global response after test execution.

Test set up	Drop weight kg	Drop height mm	Target kinetic energy J	Response after impact*
M80H450	81	450	357.57	Full collapse
M75H450	74.35	455	331.87	Full collapse
M74H200	74.35	205	149.5	Full collapse
M60H200	60.93	205	122.53	Mechanism (3R +1S)
M60H250	60.93	255	152.45	Mechanism (3R +1S)
M60H200	60.93	305	182.26	Mechanism (3R +1S)
M60H400	60.93	405	242.11	Remains intact
M60H700	60.93	705	421.43	Mechanism (1R+1S)

* The responses are from visual observation after impact before lifting the drop weight (R indicates a Rotation hinge and S indicates a Sliding release). Mechanism indicates that the onset of a mechanism was formed but the arch did not collapse

Details of the test execution are given in Fig.6. The construction was carried out using the formwork that had been used for the quasi-static test. The back seat units (abutments) were set with a separation of 1.2m inside the apparatus and then tied together horizontally using reinforcement bars. Once the abutments were constructed, the arch ring was constructed by stacking each unit one by one up to the key stone. Finally the key stone was driven in between

the right and the left rings of the arch, see Fig.6a. After the construction of the arch, a Photron camera (Fig.6b) was positioned by focusing on the units at the quarter span (just below the impactor point). The camera was placed on the same level as that of the units under focus. Finally drop weights were positioned at different heights as shown in the Fig.6c. Photographs of the arch specimens were made before and after impact to capture failure mechanisms of the arch. The deformation progress of the arches below the loading point was captured using the high speed photo camera, as shown in Fig.6d,e.

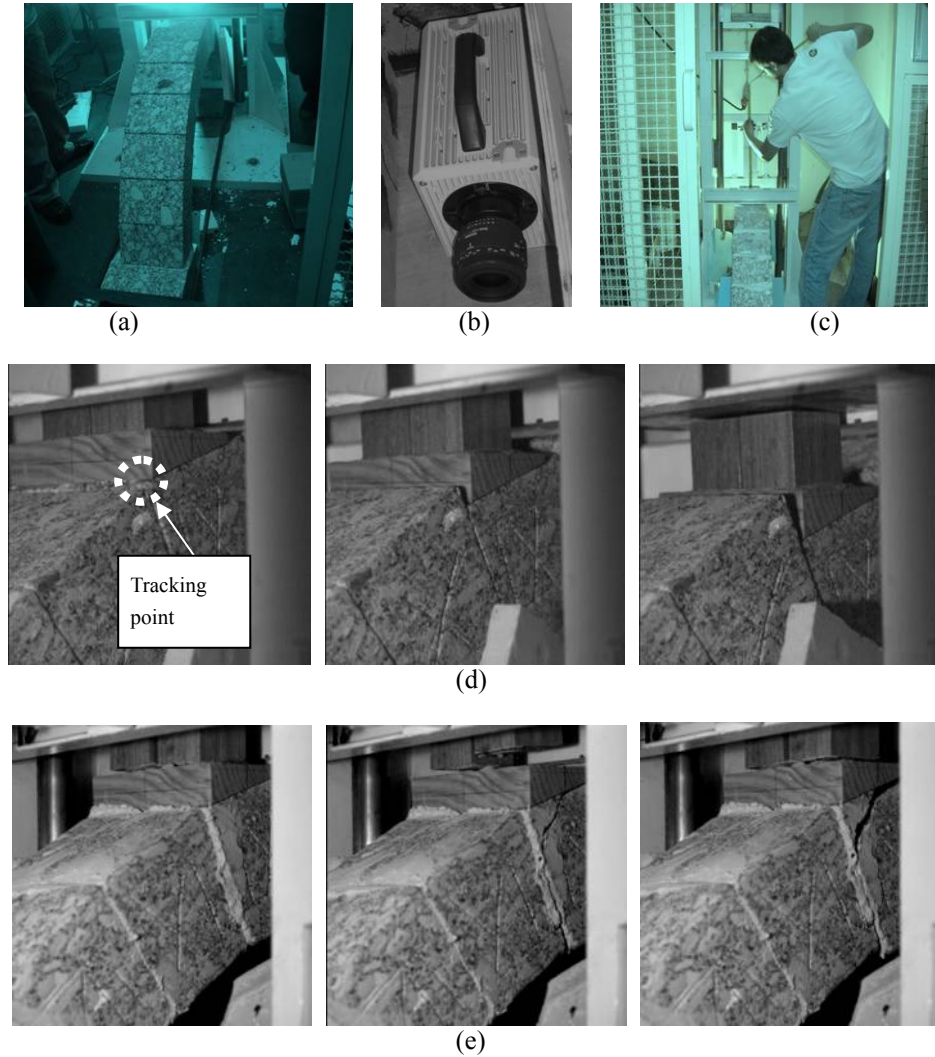


Figure 6 : Test execution: (a) assembled arch inside the drop weight; (b) Photron camera; (c) adjusting the height of the drop weight; (d) a result for a dry joint arch at 0, 100 and 700ms; (e) final result for the mortared joint arch at 0, 100 and 700ms, with damage of the stone under the load application.

3.2 Discussion

The kinematic response parameters at the control point were extracted using a tracking software (TEMA). To transform the data from units of pictures (pixels) to units of length, a scaling factor was established from the vertical dimension measurements of the wooden wedge.

The dry jointed arch was found to absorb a maximum kinetic energy of 183J whereas the mortared jointed arch resisted an impact with target energy of 421J as shown in Table 1. Fig.7 shows typical ultimate deformation of the arch specimens after impact test, with fully collapsed to the ground arches and arches forming a mechanism similar to the one observed in the quasi-static test. The kinematic response parameters of the tracking point are shown in Fig.9 for the typical dry joint and mortared joint masonry arches (M60H300 and M60H700). The

mortared joint arch rebounds back after impact with a low residual deformation, 5mm. whereas the dry jointed arch attains its static equilibrium position with permanent deformation of 60mm. For other arch specimens, please refer to Tibebe.



Figure 7 : Examples of configuration after testing: (a) M80H450 (collapse); (b) M60H300 (mechanism).

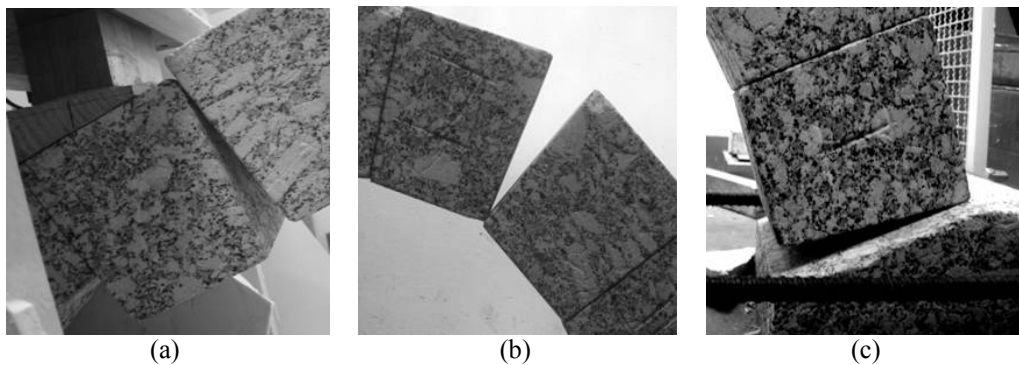


Figure 8 : Examples of releases formed after testing (M60H300): (a) left quarter span; (b) right quarter span; (c) abutment.

To validate the numerical model parameters, a calibration of the experimental and numerical results was made for the specimen M60H300 (dry masonry arch subjected to mass of 60.9kg dropping from a height of 300mm). The failure mechanism of the dry masonry arches observed during the experimental test was identical to that of the FE nonlinear explicit dynamic analysis using LS DYNA as shown in the Fig.10a. Three rotational hinges and one sliding release were prominently observed at the same location as that of the numerical simulation. Two rotational hinges centred at the extrados and intrados of the left and right quarter span, one rotational hinge at the right arch-abutment interface, and one sliding hinge at the left quarter span (impactor point). Overall, there is good agreement between the computed and observed failure mode.

As discussed above, the joint stiffness parameters influence the result of the numerical model, and hence a parametric study was done to calibrate the numerical result with that of the test. In the impact response analysis, it was found that the joint stiffness greatly affects the impulse duration (temporal distribution) of the velocity response.

4 CONCLUSIONS

Experimental and numerical studies have been conducted to investigate the performance of stone masonry arches at high rate loading resulting from explosions. A prototype scaled arch was designed and used for both numerical and experimental simulations. The capacity curve of stone masonry arches has been assessed by experimental testing, advanced non-linear analysis using DIANA and simplified methods using RING software. The behaviour of stone masonry bridges under high rate loading has been discussed using drop-weight tests on a scaled stone masonry model, and further analyzed using a non-linear explicit dynamic analysis software (LS-DYNA). It has been found that the non-linear FE simulation based on discrete masonry

material models is highly dependent on linear elastic stiffness of the masonry joints, for the particular arch considered. In the case of impact responses, it has been again found that the numerical model is highly sensitive to the joint stiffness parameters. Finally, mortared joint arches are found more robust than dry jointed masonry arches to resist impact loads.

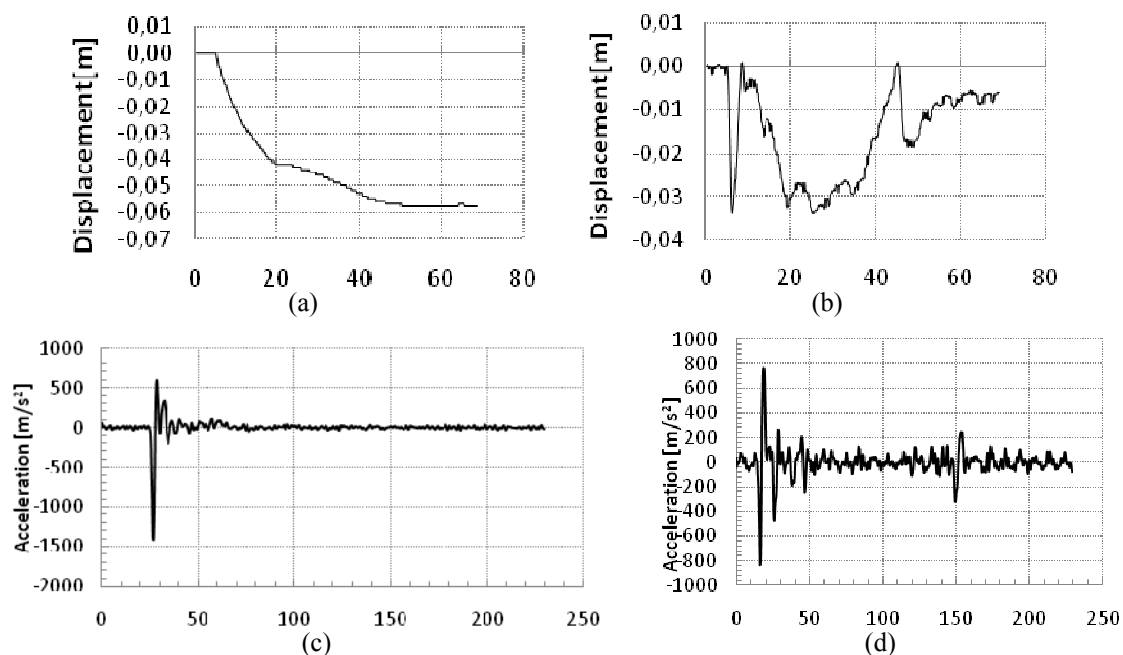


Figure 9 : Responses after impact of dry joint masonry arch (a,c) and mortared joint masonry arch (b,d), for vertical displacement and acceleration at quarter span, respectively. All horizontal axes indicate the time (ms).

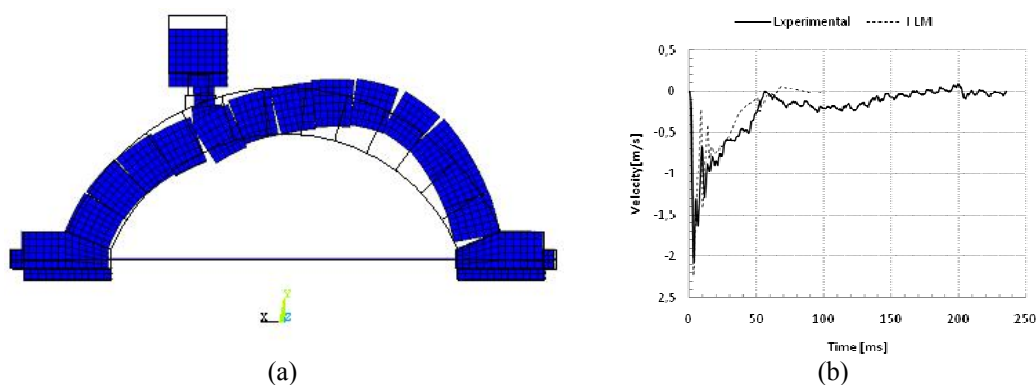


Figure 10 : Numerical results for M60H300: (a) deformed mesh at the end of the analysis; (b) comparison between numerical and experimental results.

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